Stress Distribution in Welded Flange-Bolted Web Connections

by

Garth B. Berninghaus

A thesis submitted in partial fulfillment
of the requirements for the degree of

Master of Science in Civil Engineering

University of Washington

1995

Approved by Charles W. Roeder
(Chairperson of Supervisory Committee)

Program Authorized
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Chapter 1: Introduction

1.1 Background

In the design of a building it is necessary to satisfy serviceability requirements to avoid functional failure. In buildings which may be subjected to earthquakes, serviceability requirements impose the need to keep the response of the structure in the elastic range and to limit the story drift under moderate earthquakes which may occur several times during the life of the structure.

In the case of an extreme earthquake, structural failure of the building must be prevented. This imposes additional demands on the ability of the structure to absorb and ultimately dissipate (by structural and non-structural action) energy input into the structure by ground motion. The manner in which these energy requirements are met varies with the structural system used. The system most widely used, in seismic regions, for buildings up to about 25 stories is the unbraced, moment-resistant multistory steel frame.

In general, multistory unbraced steel frame buildings are designed to respond elastically to gravity, wind, and earthquake loads as specified in various building codes. But buildings designed for code seismic forces must be expected to undergo cyclic deformations well into the inelastic range during a severe earthquake [1]. For such situations the amount of energy that the structural system can dissipate through inelastic deformation becomes the controlling design factor.

To avoid structural failure under an extreme earthquake, every member and connection of a moment resistant frame has to be designed such that its ductility allows a redistribution of moments without failure of the member or connection. Moment-resistant frames achieve ductile response through flexural yielding of the beams or shear yielding of the column panel zones. The beam-to-column connections must be capable of developing and maintaining the strength of the beams or panel zones as they undergo large cyclic inelastic deformations.

One of the most widely used moment connection details in current U.S. practice for seismic-resistant framing is the welded flange-bolted web detail shown in Figure 1. The beam web is bolted to a single plate shear tab that is shop welded to the column flange, and the beam flanges are then field welded to the column using complete-penetration single-bevel groove welds with back-up bars. The primary purpose of the back-up bar, when used in a full-penetration weld, is to support the molten weld metal during solidification. A secondary benefit is protection from atmospheric contamination on the underside or root of the weld. Most major U.S. model seismic codes adopted this detail as a standard for ductile steel moment frames, and it has been used for many years.

1.2 Present Seismic Design Philosophy

The demands for serviceability under moderate earthquakes, and for structural safety under severe earthquakes, impose requirements on the strength, stiffness, and ductility of structural components in building frames.

Serviceability requirements are usually met by providing components with sufficient strength and stiffness to assure essentially elastic response behavior and limited story drift that prevents excessive nonstructural damage under moderate earthquakes.

An explicit design for the forces and inelastic deformations that can be expected under hypothetical severe earthquakes for a given site and building can be carried out, but such a procedure is the exception rather than the rule. It is more customary to follow a series of design recommendations that are intended to assure sufficient strength and deformation capacity to provide an adequate factor of safety against failure. Basic strength criterion for connections was stated by Johnson [2]: "The joint, together with the immediately adjoining portions of the members, must be capable of plastic hinge actions. This means that relative motion between the members must be possible at an approximately constant moment of resistance."

It is generally accepted that girders rather than columns are the elements best suited to tolerate large inelastic deformations without failure, and to absorb

and dissipate the largest share of the seismic energy imparted to the structure. This leads to the widely accepted strong column-weak girder concept in which the members are sized such that plastic hinges will occur in the girders rather than in the columns. Forcing the inelastic deformations to occur in the girders, however, places a heavy burden on the beam-to-column connections which must be sufficiently strong and ductile to permit the development of plastic hinges. Based on past experience, and supported by experimental evidence, this has led in highly seismic areas to frequent use of the welded flange-bolted web connection. The full penetration groove welds used were considered to be adequate to develop the full plastic moment, M_p, of the beam. The primary function of the beam web connection is to transfer the ultimate shear force due to gravity and seismic effects.

1.3 Problem Overview and Objectives

The Northridge Earthquake was the second most costly natural disaster in U.S. history, trailing only behind Hurricane Andrew, according to one of the country's leading structural investigators [3]. Because the epicenter was in a heavily built-up area, there was three to four times as much damage as in San Francisco during the Loma Prieta Earthquake in 1989 [3].

There is currently no accurate count of steel-building related damage, but various reports indicate that localized structural damage has been identified in over

120 buildings. A majority of the reported problems were in special steel moment frame connections that consisted of welded flange-bolted web connections. Many of these special moment frames were arranged in two-bay configurations that only partially enclosed the building perimeter. Ductile yielding was intended to occur in the beams. However, fracture in the region of this field welded connection, and some accompanying fractures of the column, occurred mostly in the vicinity of the beam bottom flange connection. Cracking was initiated in the region of the flange weld near the root at the back-up bar and then propagated into the adjacent supporting column or the beam flange weld. This primary bottom flange fracture was accompanied, in some cases due to the force redistribution, by secondary cracking of the beam web shear plate, fracture of the web plate bolts, and/or top beam flange damage. In a few cases the crack propagated entirely through either the beam or the column. There was no other observable general pattern to these moment connection failures. Reported structural damage ranged from none or relatively few to many connection fractures per floor level and included mostly low and some high-rise buildings.

Various opinions and considerations have been expressed on the observed problems. Although uncertainties about the effects of site specific ground motion persist, one possible cause of these connection fractures is the unexpected high

stress concentrations in the beam flange-to-column connection. This stress concentration rendered the complete penetration flange welds to be more susceptible under cyclic load reversals to crack propagation originating from any initial notches, inadequate weld fusion/penetration, or other imperfections, including the naturally occurring notch-like condition that results from a properly fused but left-in-place steel backing bar. Despite initial appearances of connection symmetry, the bottom beam flange weld was consistently more critical than its top flange counterpart. Several feasible explanations have been proposed for these differences: increased sensitivity from the weld root at the lower extreme fiber; welding workmanship difficulties such as beam web interference (cope holes); and the additional presence of beneficial restraint at the top in the form of the concrete floor slab. Indications are that the steel beam and column material complied with applicable ASTM requirements. However, unusually high A36 material yield strengths combined with lower tensile ultimate to yield ratios could have unexpectedly caused the beam yielding to be concentrated within the welded and coped portion of the beam.

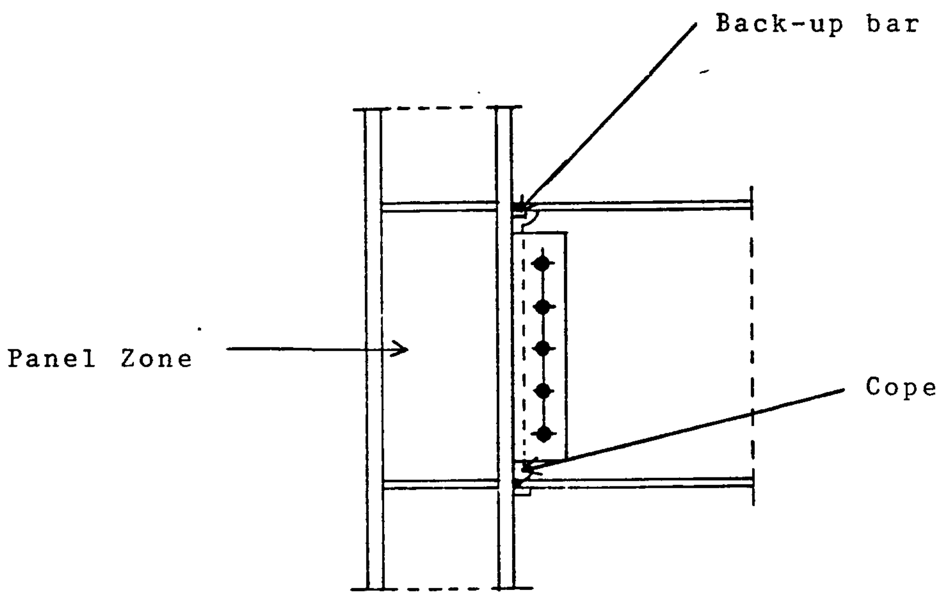

Figure 1. Welded Flange - Bolted Web Detail

Chapter 2: Research History

2.1 Shear Behavior of Joints

The shear behavior of beam-column joints has been the subject of several experimental studies conducted by Krawinkler, Bertero, and Popov in 1971 [1] and 1973 [4]. The shearing stresses in the panel zone, Figure 1, caused by lateral loading are highest at the center of the panel, with a moderate but definite drop towards the four corners [1]. When beam-column joints were stressed beyond the elastic range, yielding in the panel propagated rather slowly from the center towards the level of the beam flanges [1]. This is reflected in the load-deformational response of the joints, which exhibits an elastic range, followed by a range of gradually decreasing stiffness, and then stabilizes to a small and almost constant stiffness for a long range of deformation. The latter stiffness is a result of strain-hardening in the material. The transition range between elastic stiffness and strain-hardening stiffness is due to the fact that not only the panel zone in the joint resists the shear; the elements surrounding the panel zone also contribute significantly to this resistance. In particular, the bending resistance of the column flanges and the in-plane stiffness of the beam webs adjacent to the joint contribute to resistance. Experimental studies [1] have shown that shear yielding spreads to the corners of the panel zone usually only at large angles of distortion.

All of the joints tested in reference [4] exhibited remarkable ductility and very stable and repetitive hysterisis loops under cyclic loading. No drop in strength was noticeable even at extremely large inelastic distortions, although in some specimens with thin panel zones local buckling in the panel zone was observed. This failure mode is easily suppressed by adding column web doubler plates or diagonal panel zone stiffeners. The only detrimental effect caused by excessive joint distortions was the formation of local kinks in the beam and column flanges outside the joint as shown in Figure 2. These kinks caused high strain concentrations at the regions where the beam flanges were welded to the column, which in turn may have led to the observed fracture of material immediately adjacent to the weld or at the cope. However, this fracture only occurred after several load reversals at extremely large joint distortions in the tests reported.

In all specimens tested, [1] and [4], the post-yield stiffness and strength differed remarkably from specimen to specimen. These same tests also showed that the post-yield strength and stiffness of joints depends on the stiffness of the elements surrounding the panel zone, primarily the flexural stiffness of the column flanges, and the aspect ratio d_b/d_c. Experimental evidence [1] has shown that the column flanges take a larger percentage of the column axial load once the panel

zone starts to yield in shear. Clearly, this only holds true for columns in which the flanges have the necessary capacity to resist the full axial load plus eventual bending stresses in the yielded joint region. These factors, column flange thickness and aspect ratio d_b/d_c, as well as the stiffness of the beams and column outside the joint area, strongly affect the extent of yielding in the panel zone. The tests conducted, [1] and [4], showed that when beam-column joints are designed with adequate column flange stiffness and relatively low aspect ratio d_b/d_c, beam-column joints provided excellent energy dissipation.

2.2 Supplemental Web Welds

The performance of the welded flange-bolted web connection under cyclic loading was studied by Tsai and Popov [5] in 1988 using large-scale test specimens. In general, all specimens carried loads well above the 36 ksi nominal yield strength of the beams. Welding procedure and workmanship were shown to be critical in the performance of the connection. Variability in the ductility of the connection was observed in many circumstances and was attributed to the bolted web connection's limited ability to transfer bending moment, resulting in excessive demands on the beam-flange connections. As a result of this concern, U.S. model seismic codes; [6], [7], [8]. and [9] require that in addition to web bolts, supplemental welds be provided between the beam web and the shear tab for

beam sections with $Z_f/Z_t \leq 0.70$ as shown in Figure 3. Z_t is the beam's total plastic section modulus, and Z_f is the plastic section modulus of the beam flanges only. The supplemental web welds must be designed to develop at least 20% of the beam web's flexural strength. These provisions recognize that the web connection must be capable of carrying not only shear force, but also a portion of the beam's bending moment. These new provisions are based on rather limited data, and according to reference [6], were chosen "until additional data are available."

2.3 Ductility/Plastic Rotation Capacity

For seismic applications, the ductility developed in the beam or panel zone prior to connection failure is a critical measure of performance. Various measures of ductility have been used in the past, including plastic rotation angles and energy dissipation. Whatever measure of ductility is chosen, an estimate of the actual deformation demands on the connections in buildings during earthquakes is needed. Unfortunately, this information is rather limited.

In 1993 an experimental investigation was undertaken by Engelhardt and Hussain [10] to collect additional data on the effect of the Z_f/Z_t ratio and the web connection detail. Their specific objective was to determine if relaxation of the supplemental web welding requirements could be justified based on additional experimental data. Results were presented in the context of plastic rotation

capacity and a value of 0.015 radian was chosen as a reasonable estimate of beam plastic rotation demand in steel moment-resisting frames subject to severe earthquakes. It should be noted that actual beam plastic rotation demands are quite sensitive to frame design assumptions, frame inelastic modeling assumptions, and the chosen earthquake records. Thus, the choice of 0.015 radian was based on considerable judgment and interpretation of available data. Past experimental programs [11] and [5] appear to have used similar levels of plastic rotation to evaluate performance of specimens. It should also be noted that at joints in which the column panel zone can effectively take part in developing inelastic deformations, the plastic rotation demand on the beam is substantially reduced.

Based on the criterion of 0.015 radian of beam plastic rotation at the connection measured from the original undeformed position, the performance of the eight specimens tested in reference [10] was unsatisfactory overall. Only one of the eight specimens achieved 0.015 radian of plastic rotation. All other specimens were judged as marginally acceptable, with plastic rotations varying from 0.009 radian to 0.013 radian. Although the test results showed some effect of web-connection details on strength, there was no clear evidence from the test data on the influence of the Z_f/Z_t ratio or the web connection details on ductility. Overall, variability in the performance of the beam flange welds appears to have

had a much greater influence on plastic rotation capacity than Z_f/Z_t ratio or web-connection details. It is important to note that all test specimens in this program were constructed by a commercial steel fabricator, all welders were certified, and all groove welds were ultrasonically tested by an independent welding-inspection firm. Nonetheless, a number of welds were found to be inadequate when the connections were tested to destruction.

2.4 Objectives

Past experimental evidence, outlined in Sections 2.1 through 2.3, has shown that, when properly designed, beam-column joints provide excellent energy dissipation qualities. These sections have also shown that the flange weld area is used to transfer the majority of the beam moment to the column and is thus an area of high stress concentrations. As a result, when any section or component of the joint is improperly designed or fails to perform in the expected manner, the flange weld area is forced to carry even more of the load resulting in an even higher stress concentration.

This study was initiated to develop a computer model to analyze the stress state of a typical welded flange-bolted web connection. Various parameters within the model could then be varied in an attempt to determine and quantify which parameters most greatly affect the stress concentrations in the flange weld area.

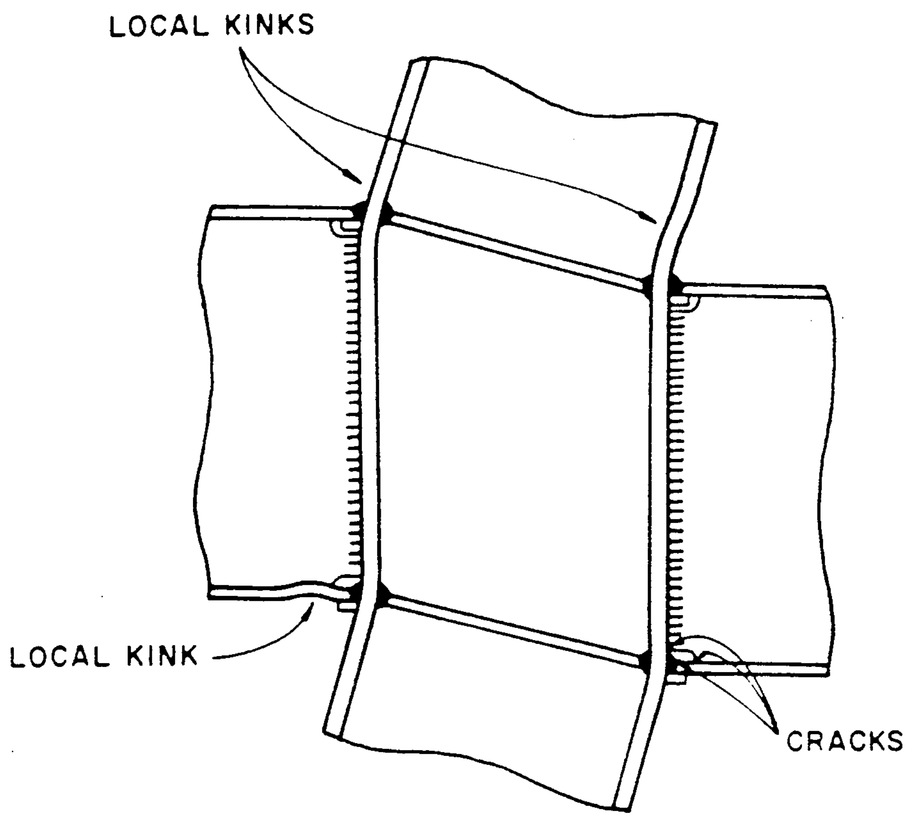

Figure 2. Local Kinks Outside Joint

Chapter 3: Development of Computer Models

3.1 Local Model

Computer models were developed to facilitate investigation into the high stress concentrations found in the bottom beam flange weld. The SAP90 [12] finite element structural analysis program was used to construct these local models. No global model was used because the goal of this study was not an analysis of one particular structure, but rather the goal was to investigate general trends in the connection itself as several parameters were varied. Static, linear, analyses were used to accomplish this goal.

3.2 Model Geometry

The model constructed is shown in Figure 4. Shell elements were used for the entire model with both membrane behavior and plate-bending behavior being included in the analysis. Mesh sizes ranged from 1/2" to 3" depending on the region under consideration. In regions at or near flange connection points, the finer mesh was used while the mesh became more coarse as the distance from the flange connection points increased. All elements used were four node elements. Three node elements were avoided because of the resulting decrease in accuracy of stress output results.

Member sizes used were W12 x 136 for the column and W24 x 55 for the beam. The erection clip was modeled as a 1/2" thick steel plate. The gap between

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the beam web and column flange was modeled as 1/2". The member sizes and geometry matches typical specimens used in reference [10].

The complete-penetration, single-bevel, groove welds were modeled by connecting the beam flange elements to the column flange elements. The welded portion of the erection clip was also modeled by direct connection of erection clip elements to the appropriate column flange elements.

The erection plate bolts were modeled by using the constraints feature of the program which allows the user to specify that joints may be constrained such that they have the same translations and rotations in the X, Y, and Z directions. Six bolts were used to attach the erection clip to the beam web which resulted in six pairs of constrained joints. Only the translations were constrained while the rotations were left free.

Copes or cope holes are used to allow access for the welder as shown in Figure 1. Dimensions are given in terms of a cope radius which is a function of the beam web thickness as specified in the American Welding Society (AWS) code [13]. For the W24 x 55 beam used in all models, the minimum allowable cope radius is .395". There are no upper limits on cope size. Copes were modeled by omitting elements in the appropriate locations of the beam web as shown in

Figure 5. This resulted in a square or rectangular cope rather than the typical circular one.

Column stiffeners were modeled as rigidly connected (welded) on all three sides of contact. They extended to the full width of the column flange and were symmetrical about both sides of the column web for a total of four stiffeners. They were located in the same horizontal plane as the beam flanges.

3.3 Loading

The model was loaded as shown in Figure 6. The relative magnitude of each load was determined by normalizing the beam moment (M_b) to equal the elastic section modulus of the beam. In anticipation of further work with this model in which the beam size would be changed, this normalization was chosen to facilitate comparison between models. In this case, the beam moment was equal to 114 k-in. Typically beam shear (V_b) for this type of connection falls within the range of $1/10(M_b/d_b)$ to $1/5(M_b/d_b)$. A mid-range value of $2/15(M_b/d_b)$ was selected. Equilibrium calculations were used to determine the appropriate values for the column moment (M_c) and the column shear (V_c). Column axial loads (P) are frequently in the range of 45% of the yield stress (F_y). To correspond with the normalization of the beam moment, a column axial load of 45% of 1 ksi was used.

All loads were applied as point loads as shown in Figure 7. The modulus of elasticity was taken as 29,000 ksi and the value used for Poisson's ratio was 0.3.

3.4 Parameters Varied

A total of 15 models were developed, as shown in Table 1, and tested in this study to determine the effect of various parameters on the stress distribution. Each model varied only one parameter from the original model (Model 1) so that trends or changes in the beam flange stress distribution would be easily attributed to the appropriate variable. Models within a given parameter grouping could then be compared with one another as well as the original model (Model 1).

Models 2, 3, 10, 11, and 12 were developed to determine if the size of the cope had an effect on the beam flange stress distribution. Model 1 had no cope, and it only had the constant 1/2" gap between the beam web and column flange. Model 2, which represented a typical coped connection, had one shell element removed at the bottom of the beam web corner and one shell element removed at the top to model the cope holes. Figure 8 shows the elements removed in Models 2, 3, 10, 11, and 12.

Models 4, 5, 14, and 15 were created to investigate effects of variation in the thickness of the column flange. Model 4 had a column flange thickness equal

to 50% that of Model 1 while Models 5, 14, and 15 had respective column flange thicknesses equal to 200%, 75%, and 150% that of Model 1.

Models 6 and 7 were used to check for changes in stress distributions caused by variation in the column web thickness. Model 6 had a column web thickness equal to 50% that of the Model 1 column web thickness while the Model 7 thickness was 200% that of Model 1.

Models 8, 9, and 13 were used to check for any effects of adding web stiffeners (continuity plates) to the column web. Model 8 used column web stiffeners with thicknesses equal to 50% of the beam flange thickness while Models 9 and 13 used 100% and 200% respectively.

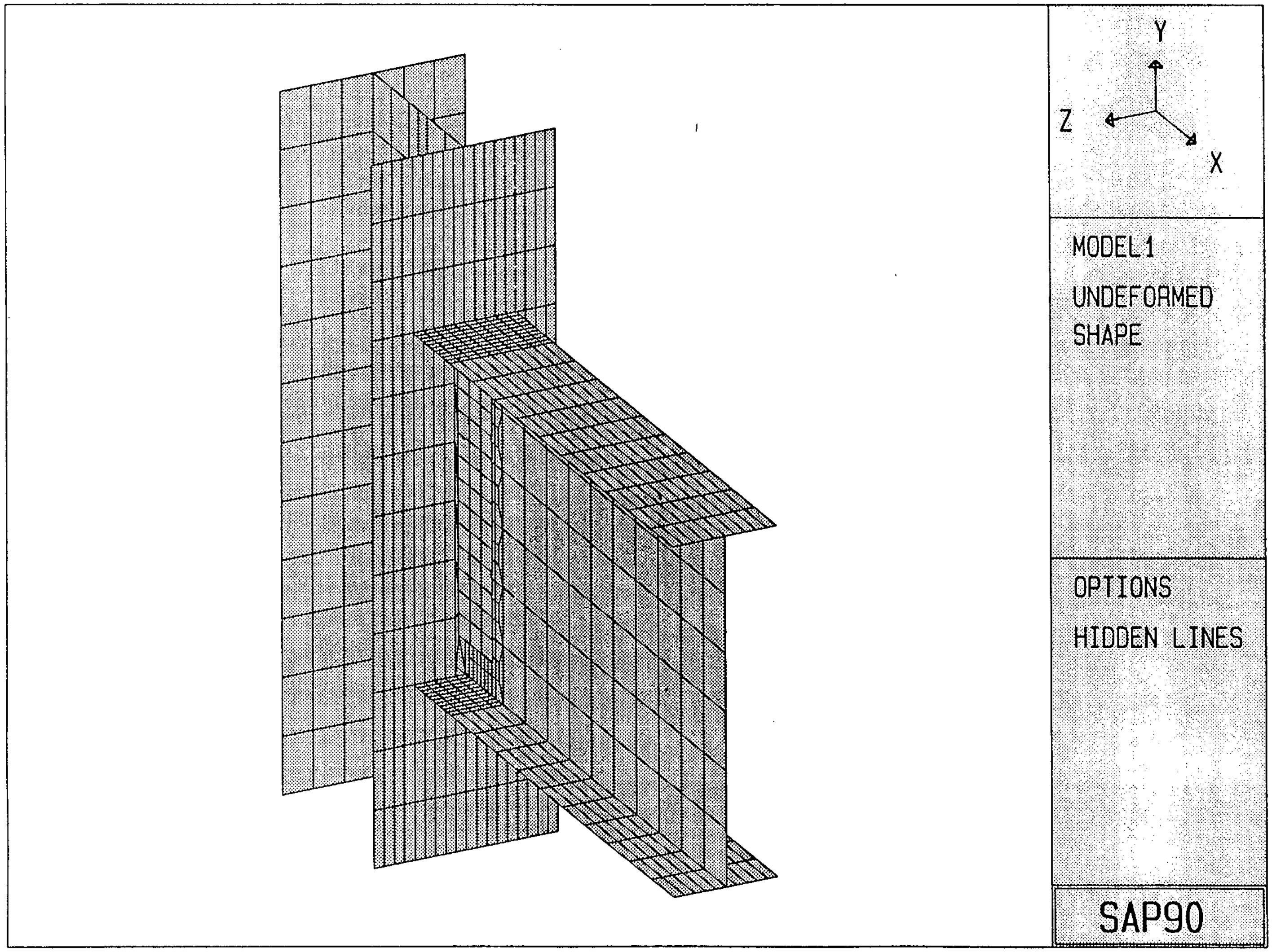


Figure 4. SAP90 Model

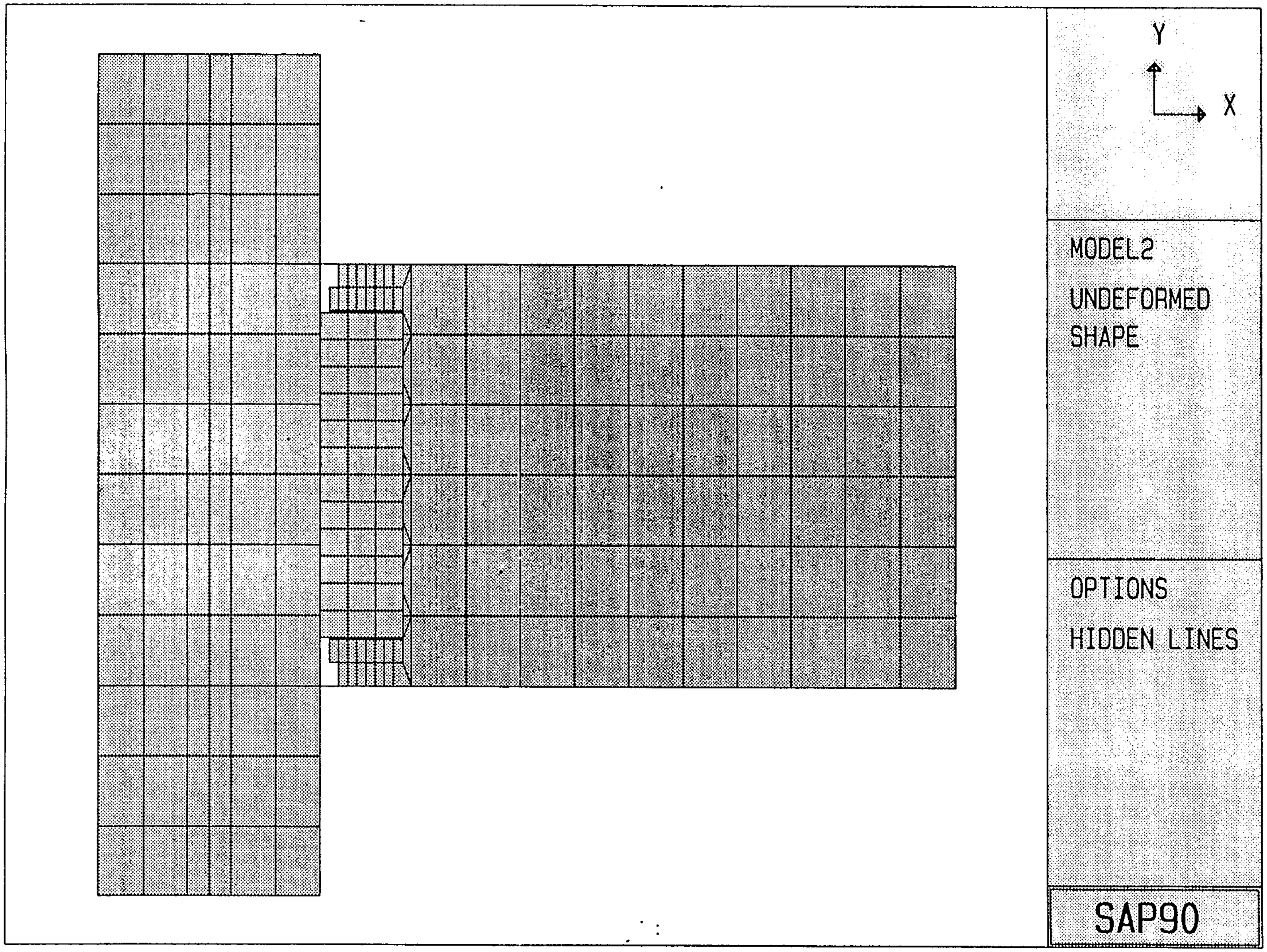
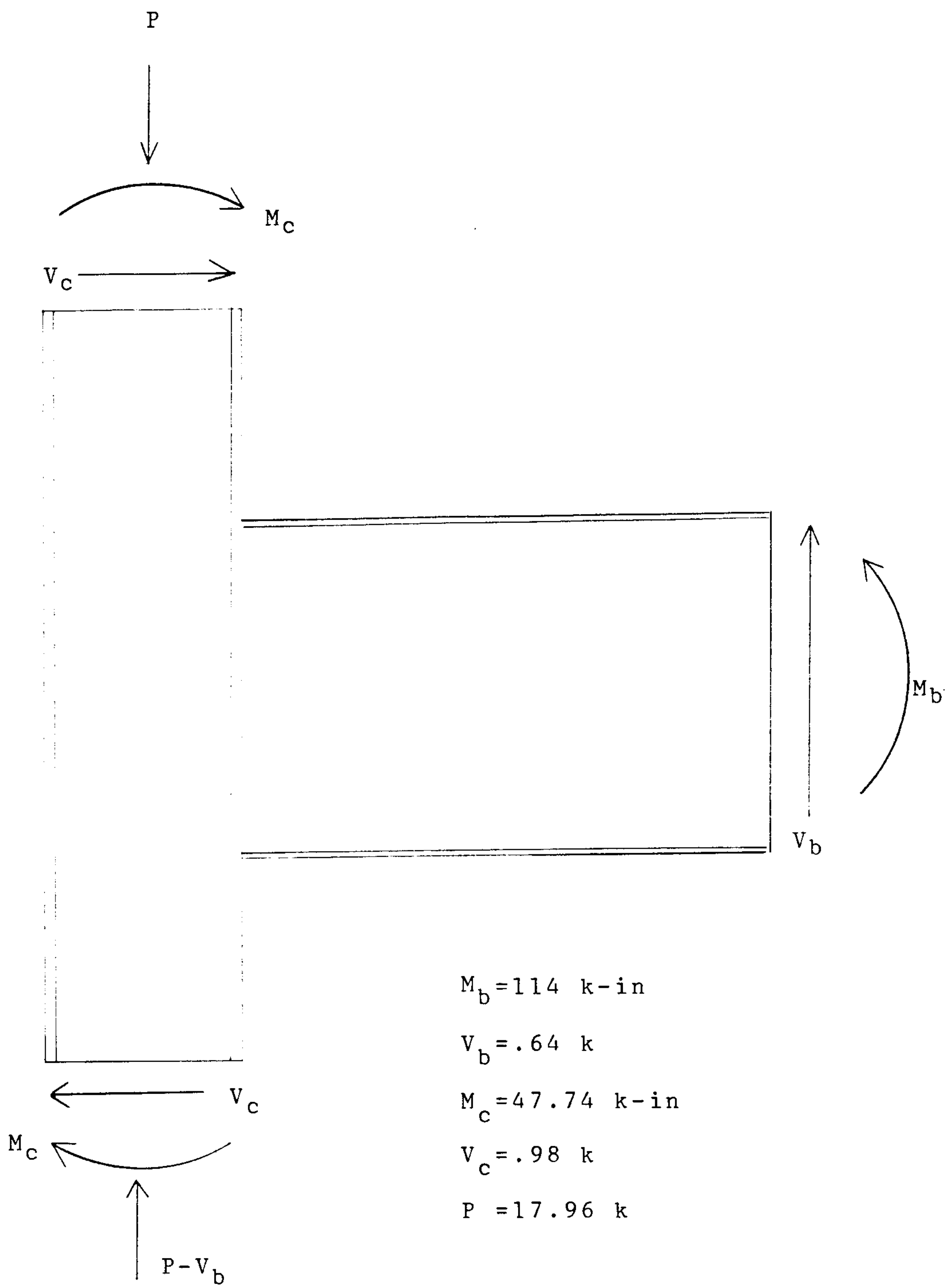


Figure 5. Cope Elements Omitted



M_b=114 k-in

V_b=.64 k

M_c=47.74 k-in

V_c=.98 k

P =17.96 k

Figure 6. Loading Scheme

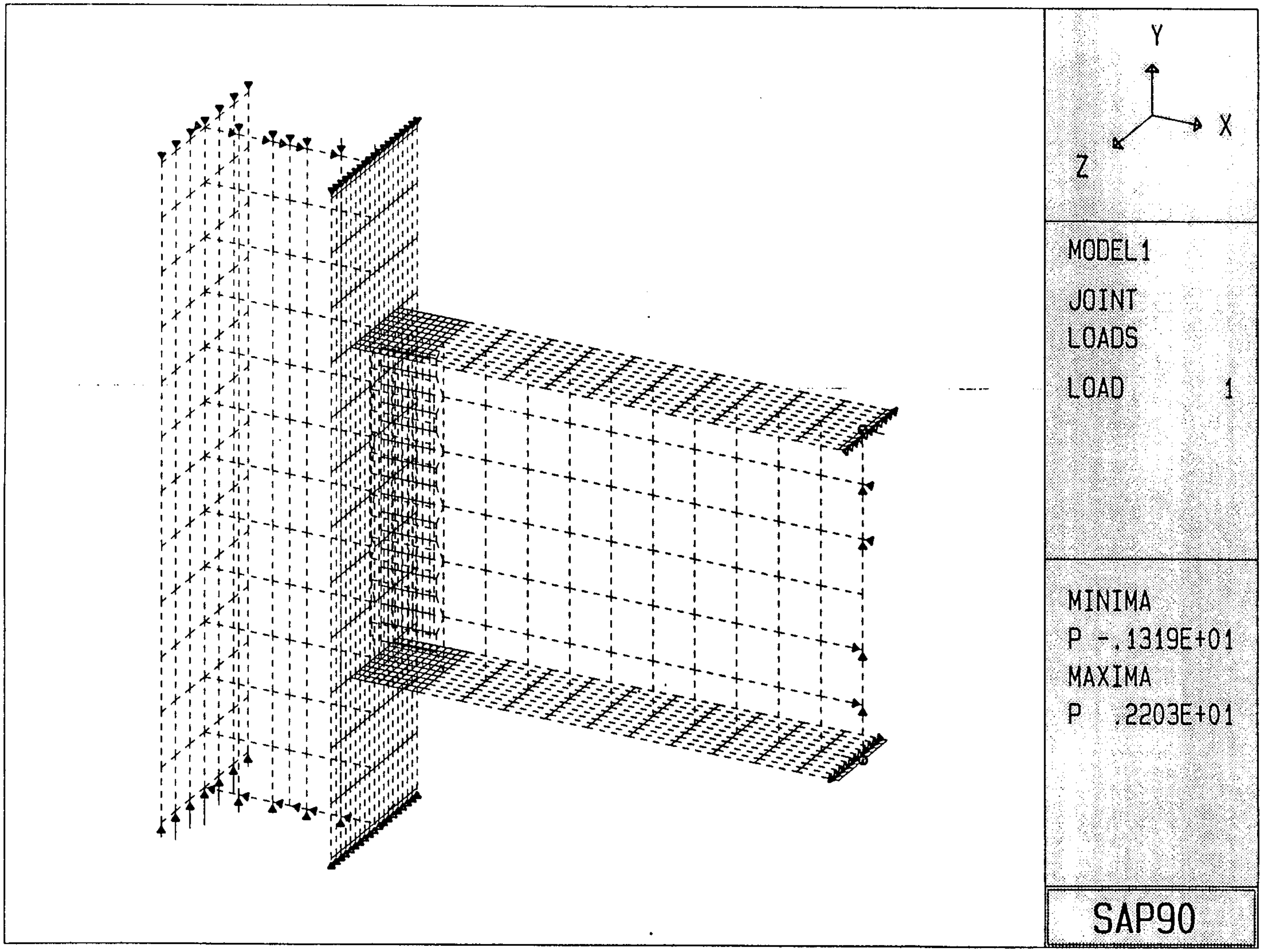
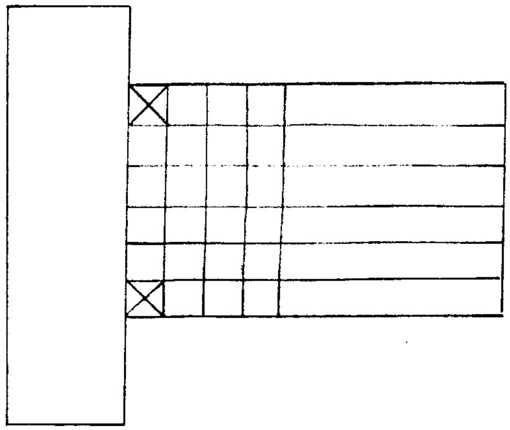


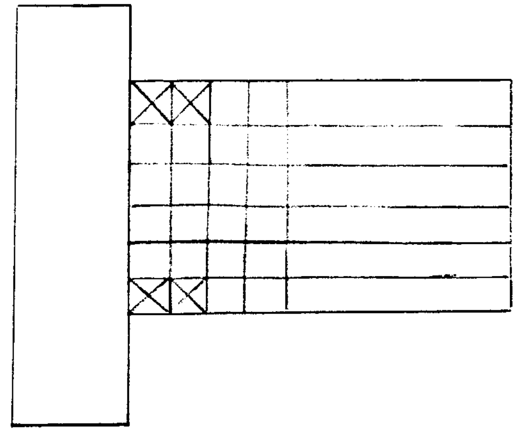
Figure 7. Load Application Points

Table 1. SAP90 Models

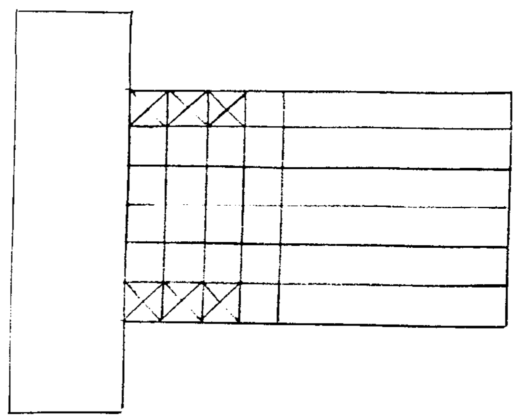
Model Number	Description
1	Original
2	One cope element removed
3	Two horizontal cope elements removed
4	Column flanges 50% of Model 1
5	Column flanges 200% of Model 1
6	Column web 50% of Model 1
7	Column web 200% of Model 1
8	Column stiffeners with thickness 50% of beam flange
9	Column stiffeners with thickness 100% of beam flange
10	Three horizontal cope elements removed
11	Two vertical cope elements removed
12	Two vertical and two horizontal cope elements removed
13	Column stiffeners with thickness 200% of beam flange
14	Column flanges 75% of Model 1
15	Column flanges 150% of Model 1



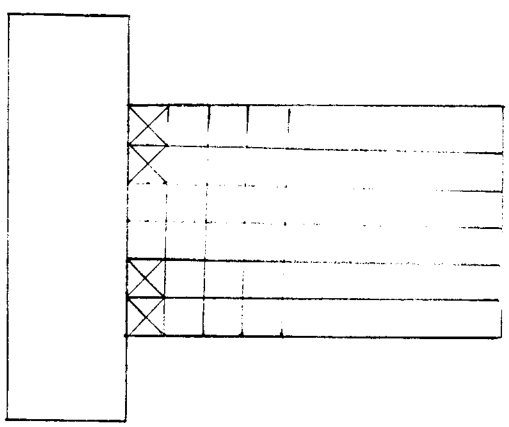
Model 2



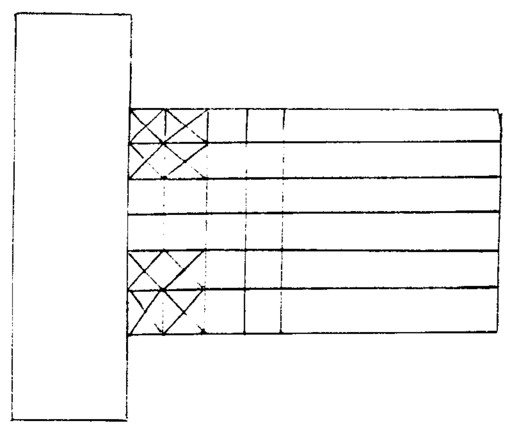
Model 3



Model 10



Model 11



Model 12

Figure 8. Cope Models

Chapter 4: Results of the SAP90 Computer Analysis

4.1 General Discussion

When the stress output option is chosen in a SAP90 analysis, the output for shell elements is given in the form of top and bottom stresses. "Top" and "bottom" are defined by the assignment of global axes and shell thickness.

Plate bending behavior dominates at the extreme fiber of the beam flange and becomes less pronounced as one moves along the beam flange away from the joint. In an effort to normalize the results so that a comparison of beam stress concentrations was possible regardless of relative position from the groove weld, the top and bottom stresses at all element joints were averaged.

SAP90 stress output yields both maximum and minimum principal stresses, shear stresses, and normal stresses oriented along the longitudinal and transverse axes of the beam. The normal stress oriented along the longitudinal axis of the beam best illustrates the variations in stress along the beam flange and is denoted as the S11 stress. These S11 stress results were averaged as discussed in the previous paragraph and the results entered into a spreadsheet as shown in Table 2.

Four positions of interest were chosen to analyze any change in S11 stress concentrations along the beam flange as the distance from the joint was increased. The first was at a position with an x coordinate equal to 12.16". This location corresponds to the point of contact between the beam and column webs, the

location of the full-penetration groove weld. The three remaining positions were located at X = 13.66", 15.16", and 20.16". These represent distances from the point of flange contact equal to 1.5", 3.0", and 8.0" respectively. The expected stress was calculated at each of these four points of interest by use of the following equation: Expected stress = $(S_x + V_b(e))/S_x$ where S_x = Elastic section modulus of the beam, V_b = Beam shear, and e = Eccentricity from the beam end. This expected stress was then plotted as a horizontal, dotted line on each stress graph.

Displacement data was also tabulated in the longitudinal axis direction of the beam (X direction) at the same four positions. No averaging was necessary since this output data is given in terms of global displacements at the joint rather than at the top and bottom of the shell element. The displacement results for Model 1 are shown in Table 3.

Figure 9 illustrates the deformed shape for Model 1. Since this was a static analysis, all models exhibited the same relative deformed shape; that is, top flange in compression and bottom flange in tension.

4.2 Column Stiffener Models

Models 8, 9, and 13 were created to investigate the effect on the beam flange stress distribution when column web stiffeners are added. As mentioned previously, Model 1 was created as the reference point for all models and

therefore, it had no stiffeners. Model 8 used stiffeners with thicknesses equal to half the thickness of the beam flange. The stiffeners used in Model 9 were equal in thickness to the beam flange while Model 13 stiffeners had thicknesses equal to twice the thickness of the beam flange.

Figure 10 illustrates the bottom flange stress distribution results of the stiffener models along with the results of Model 1. All models are identical except for the variation in stiffener thickness. The key shows the progression of models from no stiffeners present (Model 1) to thickest stiffener (Model 13). The X axis of the plot gives the relative location along the width of the beam flange. Location 6 is at the beam flange midpoint while positions 1 and 11 are at the edges. The general trends among the individual models are as one would expect with the highest stress being at the middle of the flange. The stresses decreased as distance from the midpoint increased. In Models 8, 9, and 13, the stress increases towards the edge of the flange while in Model 1 this stress increase at the edges is not present. This is due to the fact that the column web stiffeners extend all the way to the edge of the column flange instead of stopping at the edge of the beam flange. Model 1 had no stiffeners, so this trend was not present. General trends when models are compared to one another show that thicker stiffeners result in lower stress concentrations around the center of the beam flange. Specifically, at

location 6 the stress in Model 8 is 17% less than that in Model 1, while stresses in Models 9 and 13 are lower than Model 1 by 33% and 37% respectively.

Figure 11 shows the stress distributions at a point 1.5" inches away from the point of contact between the beam and column flanges. The same trends as previously discussed are present with the exception that the stress increases at the outside edges of the beam flange have disappeared. This is easily explained by the fact that the position of interest is no longer immediately adjacent to the column stiffeners, and the stress risers found previously have now been dissipated by the additional beam flange material. A trend that is present in Figure 10 but is more pronounced in Figure 11 is that increased stiffener thickness resulted in a more uniform stress distribution. These results make sense because one of the more attractive results of using a web stiffener is that the stress is more evenly distributed over a given area.

The trends previously discussed are also present in Figure 12 but are not as pronounced since the location is now at a point 3" from the point of contact. Similarly the location which is 8" from the point of contact yields no new results or trends.

Figures 13 and 14 are the beam top flange counterparts to Figures 10 and 11 which illustrate beam bottom flange stress distributions. The trends are identical

to those previously discussed and the symmetry between top and bottom flanges is excellent. Top and bottom flange stress values differ by no more than 3%.

The displacement data was not the focus or interest of this study but rather was used as an indicator of correct model behavior in a global sense. Deflections did not exceed .000482" at the groove weld and graphical trends were similar to those described for the stress distributions. That is, the increased stiffener size resulted in lower, more uniform deflection distributions. Additionally, the deflections were greatest in the center of the beam flange and decreased as distance from the flange center increased.

4.3 Column Flange Models

Models 4, 5, 14, and 15 were created to investigate the effect on the beam flange stress distribution when column flange thickness was varied. As before, Model 1 was created as the reference point for all models and therefore, all column flange thicknesses are referenced to Model 1. Model 4 had a column flange thickness equal to 50% that of Model 1 while Models 5, 14, and 15 had respective column flange thicknesses equal to 200%, 75%, and 150% that of Model 1.

Stress and displacement data were recorded, tabulated and displayed in the same manner as described in Section 4.1. The displacement data was again used as

a global check and, in this series of models, did not exceed .00052". Displacement trends were similar to those described for deflections in Section 4.2.

Figure 15 illustrates the stress distribution results of the column flange models along with the results of Model 1. All models are identical except for the variation in column flange thickness. The key shows the progression of models from thin flange (Model 4) to thick flange (Model 5). The trends among the individual models are the same as noted and explained in section 4.2 with these noted exceptions. The stress increase towards the flange edges was still present but not as pronounced. Only models which had flanges thicker than Model 1 (Models 15 and 5) exhibited this behavior. This is consistent with the results in the column stiffener models.

General trends when models are compared to one another show that the stiffer the model the lower the stress concentration around the center of the beam flange. Specifically, at location 6, the stress in Model 14 is 21% less than that in Model 4 while stresses in Models 1, 15, and 5 are lower than Model 4 by 40%, 44%, and 48% respectively.

Figure 16 shows the stress distributions at a point 1.5" inches away from the point of contact between the beam and column flanges. The same trends as

previously discussed in section 4.1 for this location as well as those which are 3" and 8" from the point of contact are present with no exceptions.

Figures 17 and 18 are the beam top flange counterparts to Figures 15 and 16 which illustrate beam bottom flange stress distributions. As with the column stiffener models, the symmetry between top and bottom flanges is very good. Differences between top and bottom flange stresses differed by no more than 5%.

4.4 Beam Cope Models

Models 2, 3, 10, 11, and 12 were developed to determine if the size of the cope had an effect on the beam flange stress distribution. Again, Model 1 was created as the reference point for all models and therefore, all coped models are referenced to Model 1. Model 1 had no cope, it had only the constant 1/2" gap between the beam web and column flange. Model 2 had one shell element removed at the bottom beam web corner and one shell element removed at the top to model the cope holes. Figure 8, found in chapter 3, shows the elements removed in Models 2, 3, 10, 11, and 12 to model various cope configurations.

Stress and displacement data were recorded, tabulated and displayed in the same manner as with previous models. Again, the displacement data was used as a global check and, in this series of models, did not exceed .00026" which was

lowest of all model groups thus far. Displacement trends were similar to those described in previous sections and thus helped to validate model behavior.

Models 11 and 12 were grouped together since they modeled cope configurations in which not one but two rows of cope elements were deleted. Figure 19 illustrates the stress distribution results of these two cope models along with the results of Model 1. No new trends or variations in data were noted. Stress comparisons at location 6 showed that the stress in Model 11 is 13% less than that in Model 1 and the stress Model 12 is less than that in Model 1 by 19%. These comparisons with Model 1 show differences which are quite small relative to column stiffener models and column flange models which resulted in differences from Model 1 as large as 37% and 48% respectively. The small differences noted in Models 11 and 12 are compounded by the fact that this type of a cope configuration represents cope dimensions which are 30% larger than typical for this size beam. As with previous models, the stress differences at the center of the beam flange became less pronounced as distance from the point of contact increased. Symmetry between top and bottom flanges was excellent. Differences between top and bottom flange stress values differed by no more than 2%.

Models 2, 3, and 10 were grouped together since they modeled cope configurations in which only one row of cope elements were deleted. Stress

comparisons at location 6 showed results that differed from Model 1 by no more than 8%. Stress distribution results revealed no new trends or variations in data. Symmetry between top and bottom flanges was excellent. Differences between top and bottom flange stress values differed by no more than 1%.

4.5 Column Web Models

Models 6 and 7 were used to check for changes in beam flange stress distributions caused by variation in column web thickness. Model 6 had a column web thickness equal to 50% that of the Model 1 column web thickness while the Model 7 thickness was 200% that of Model 1.

Stress and displacement data were recorded, tabulated and displayed in the same manner as with previous models. Displacement data again provided a global check of the models and deflections did not exceed .00021".

Flange stress distribution results of Models 6 and 7 never differed from those of Model 1 at any location by more than 5%. Symmetry between top and bottom flanges is again excellent with differences between top and bottom flange stress values of no more than 2%.

Table 2. Stress Results: Model 1

				Bottom Flange		
Joint	S11 (ksi)	Joint	S11 (ksi)	Joint	S11 (ksi)	Joint
153	0.76995	4.13E+2	7.85E-1	4.46E+2	9.40E-1	6.61E+2
154	0.96674	4.14E+2	1.11E+0	4.47E+2	1.11E+0	6.62E+2
155	1.2943	4.15E+2	1.30E+0	4.48E+2	1.27E+0	6.63E+2
156	1.72	4.16E+2	1.64E+0	4.49E+2	1.44E+0	6.64E+2
157	2.614	4.17E+2	1.96E+0	4.50E+2	1.50E+0	6.65E+2
158	4.31	4.18E+2	2.07E+0	4.51E+2	1.46E+0	6.66E+2
159	2.242	4.19E+2	1.98E+0	4.52E+2	1.53E+0	6.67E+2
160	1.78	4.20E+2	1.69E+0	4.53E+2	1.47E+0	6.68E+2
161	1.3249	4.21E+2	1.44E+0	4.54E+2	1.36E+0	6.69E+2
162	1.0536	4.22E+2	1.17E+0	4.55E+2	1.24E+0	6.70E+2
163	0.91341	4.23E+2	7.60E-1	4.56E+2	1.16E+0	6.71E+2
				Top Flange		
Joint	S11 (ksi)	Joint	S11 (ksi)	Joint	S11 (ksi)	Joint
267	-0.75	5.23E+2	-7.82E-1	5.56E+2	-9.30E-1	7.71E+2
268	-0.92	5.24E+2	-1.05E+0	5.57E+2	-1.10E+0	7.72E+2
269	-1.25	5.25E+2	-1.21E+0	5.58E+2	-1.25E+0	7.73E+2
270	-1.7	5.26E+2	-1.60E+0	5.59E+2	-1.40E+0	7.74E+2
271	-2.63	5.27E+2	-1.90E+0	5.60E+2	-1.52E+0	7.75E+2
272	-4.37	5.28E+2	-2.01E+0	5.61E+2	-1.45E+0	7.76E+2
273	-2.2	5.29E+2	-1.93E+0	5.62E+2	-1.52E+0	7.77E+2
274	-1.7	5.30E+2	-1.67E+0	5.63E+2	-1.45E+0	7.78E+2
275	-1.3	5.31E+2	-1.40E+0	5.64E+2	-1.32E+0	7.79E+2
276	-1.1	5.32E+2	-1.10E+0	5.65E+2	-1.22E+0	7.80E+2
277	-0.9	5.33E+2	-7.50E-1	5.66E+2	-1.15E+0	7.81E+2

Table 3. Displacement Results: Model 1

				Bottom Flange		
Joint	"X" Disp.	Joint	"X" Disp.	Joint	"X" Disp.	Joint
	(in.)		(in.)		(in.)	
153	-0.000316	4.13E+2	-2.88E-4	4.46E+2	-2.51E-4	6.61E+2
154	-0.000381	4.14E+2	-3.06E-4	4.47E+2	-2.57E-4	6.62E+2
155	-0.000381	4.15E+2	-3.27E-4	4.48E+2	-2.65E-4	6.63E+2
155	-0.000419	4.16E+2	-3.47E-4	4.49E+2	-2.74E-4	6.64E+2
157	-0.000459	4.17E+2	-3.60E-4	4.50E+2	-2.78E-4	6.65E+2
158	-0.000482	4.18E+2	-3.61E-4	4.51E+2	-2.79E-4	6.66E+2
159	-0.000467	4.19E+2	-3.68E-4	4.52E+2	-2.86E-4	6.67E+2
160	-0.000439	4.20E+2	-3.65E-4	4.53E+2	-2.89E-4	6.68E+2
161	-0.000411	4.21E+2	-3.54E-4	4.54E+2	-2.89E-4	6.69E+2
162	-0.000388	4.22E+2	-3.42E-4	4.55E+2	-2.88E-4	6.70E+2
163	-0.000368	4.23E+2	-3.33E-4	4.56E+2	-2.90E-4	6.71E+2
				Top Flange		
Joint	"X" Disp.	Joint	"X" Disp.	Joint	"X" Disp.	Joint
	(in.)		(in.)		(in.)	
267	0.000329	5.23E+2	3.02E-4	5.56E+2	2.62E-4	7.71E+2
268	0.000355	5.24E+2	3.14E-4	5.57E+2	2.65E-4	7.72E+2
269	0.000385	5.25E+2	3.30E-4	5.58E+2	2.71E-4	7.73E+2
270	0.00042	5.26E+2	3.47E-4	5.59E+2	2.77E-4	7.74E+2
271	0.000458	5.27E+2	3.59E-4	5.60E+2	2.79E-4	7.75E+2
272	0.00048	5.28E+2	3.60E-4	5.61E+2	2.77E-4	7.76E+2
273	0.000466	5.29E+2	3.67E-4	5.62E+2	2.87E-4	7.77E+2
274	0.00044	5.30E+2	3.65E-4	5.63E+2	2.92E-4	7.78E+2
275	0.000415	5.31E+2	3.57E-4	5.64E+2	2.94E-4	7.79E+2
276	0.000395	5.32E+2	3.49E-4	5.65E+2	2.96E-4	7.80E+2
277	0.000378	5.33E+2	3.46E-4	5.66E+2	3.00E-4	7.81E+2

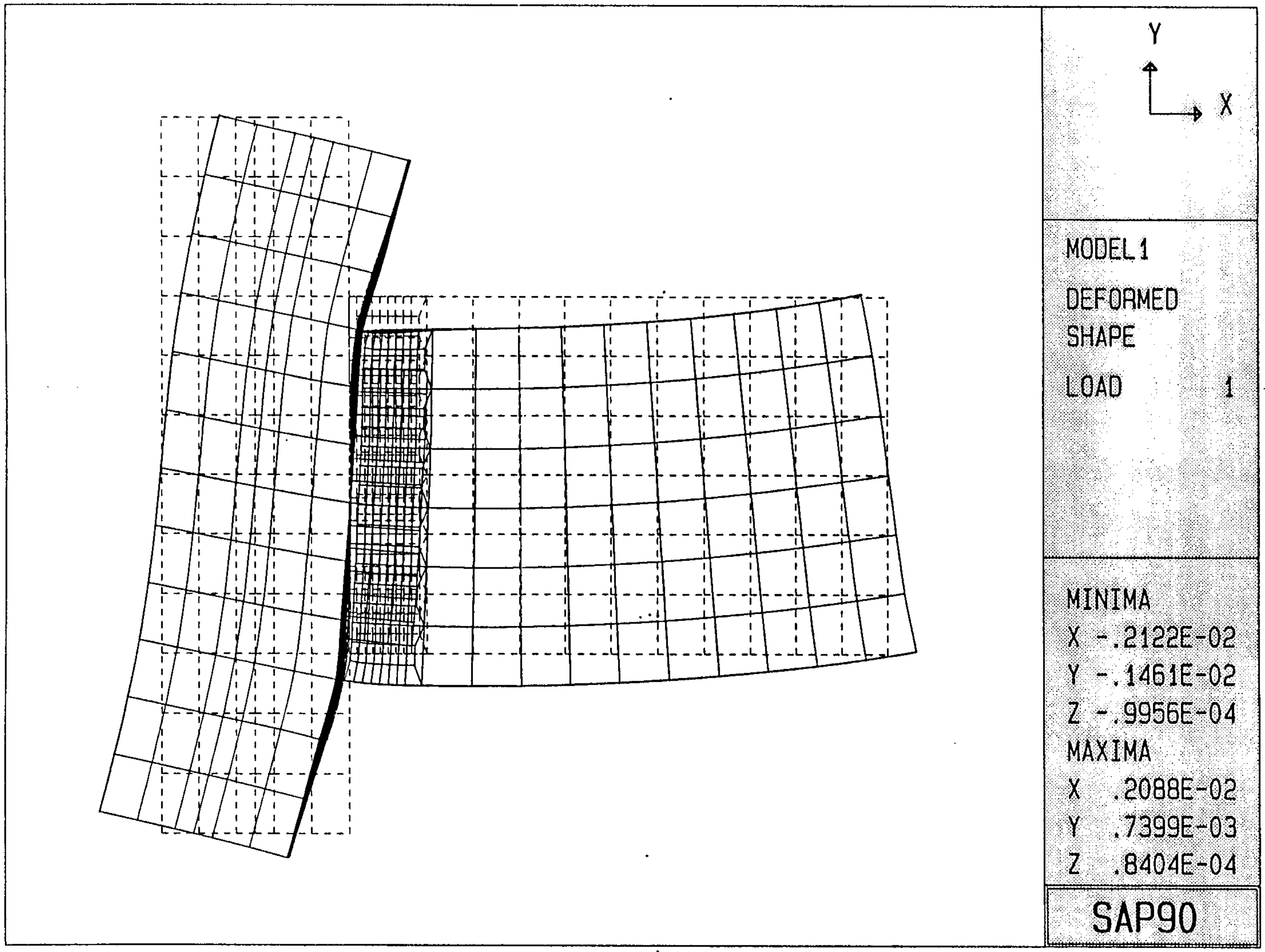


Figure 9. Deformed Shape: Model 1

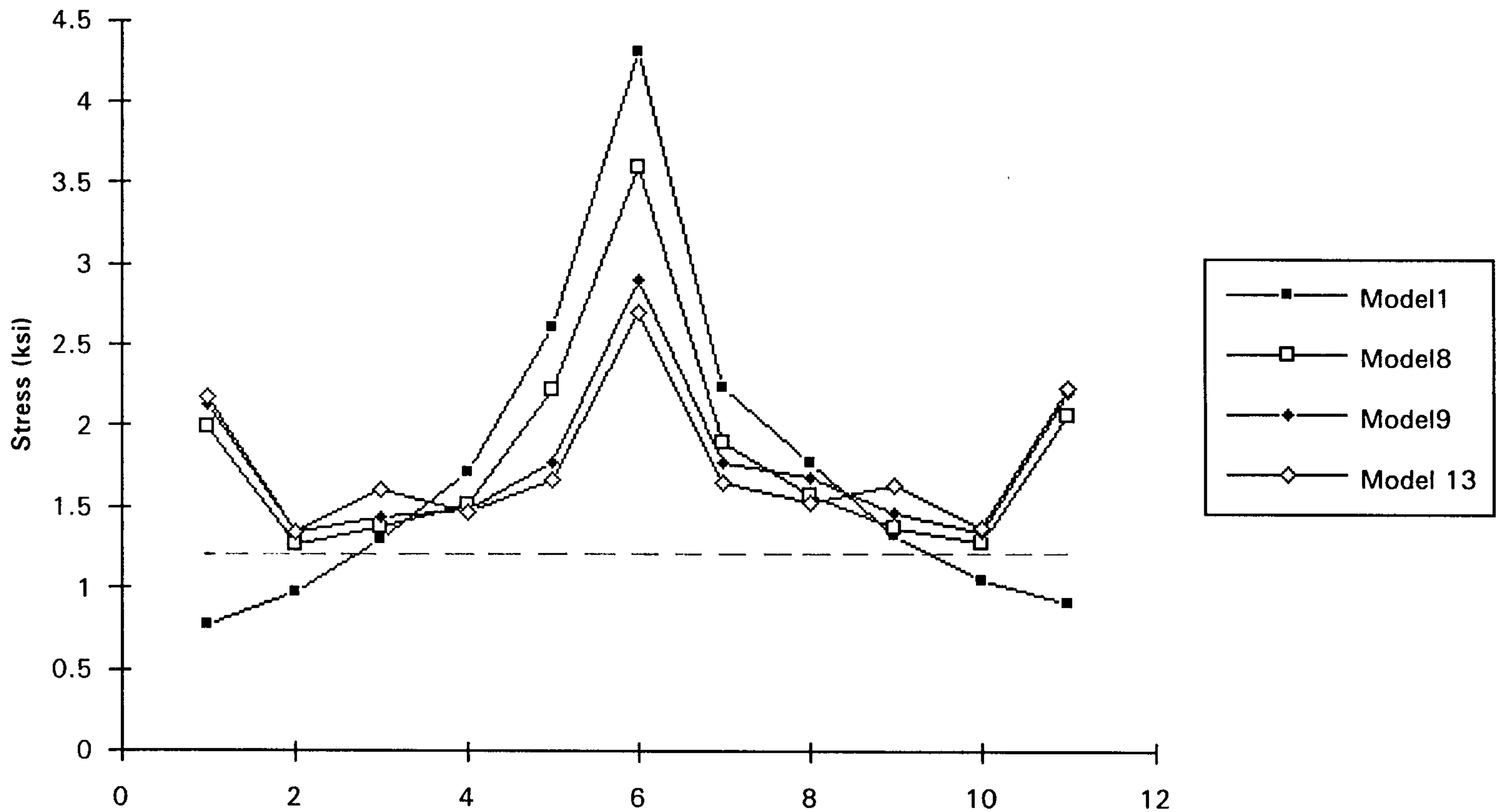


Figure 10. Stiffener Models: Bottom Flange Stress Results at Groove Weld

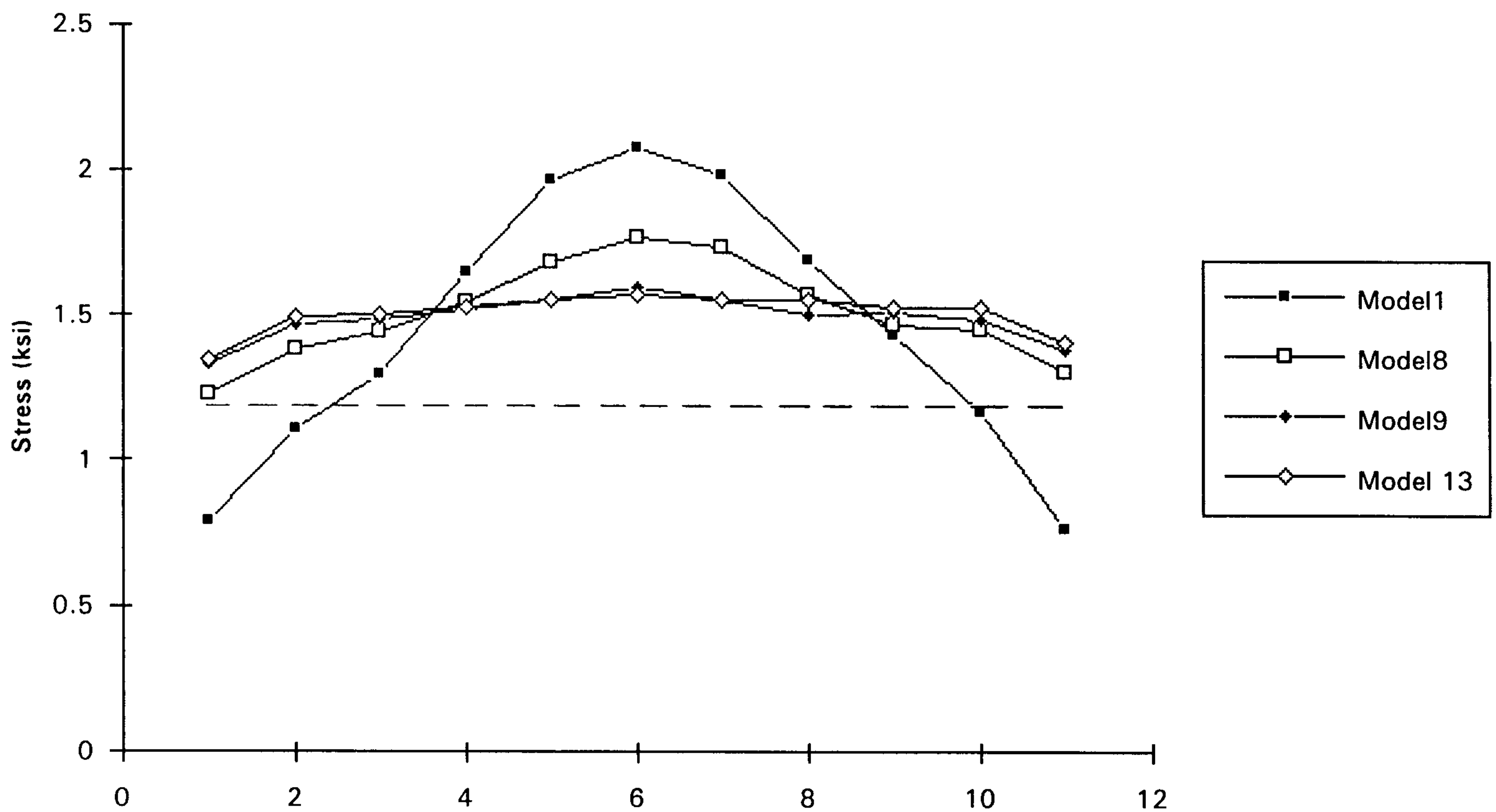


Figure 11. Stiffener Models: Bottom Flange Stress Results at 1.5" from Groove Weld

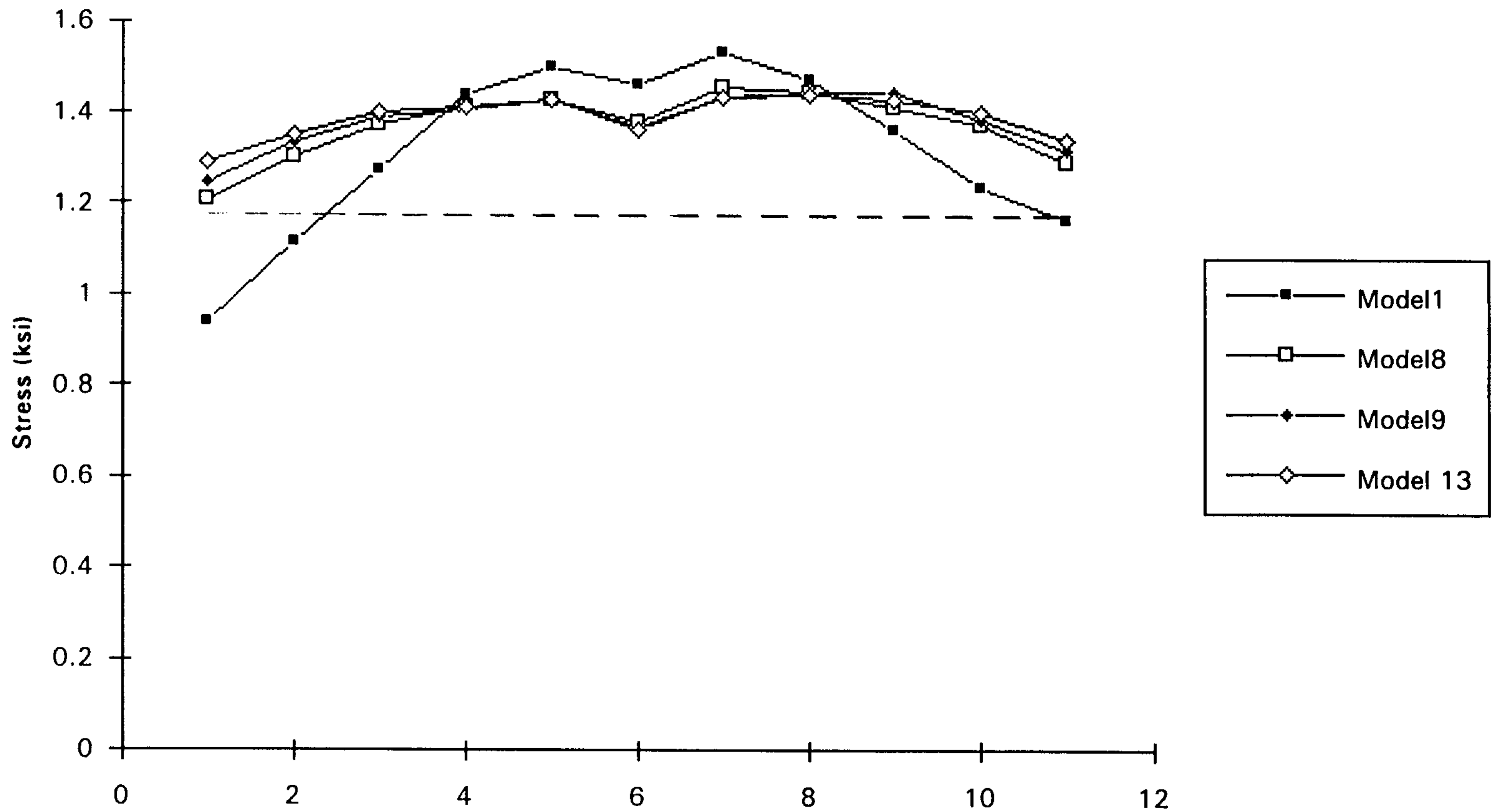


Figure 12. Stiffener Models: Bottom Flange Stress Results at 3" from Groove Weld

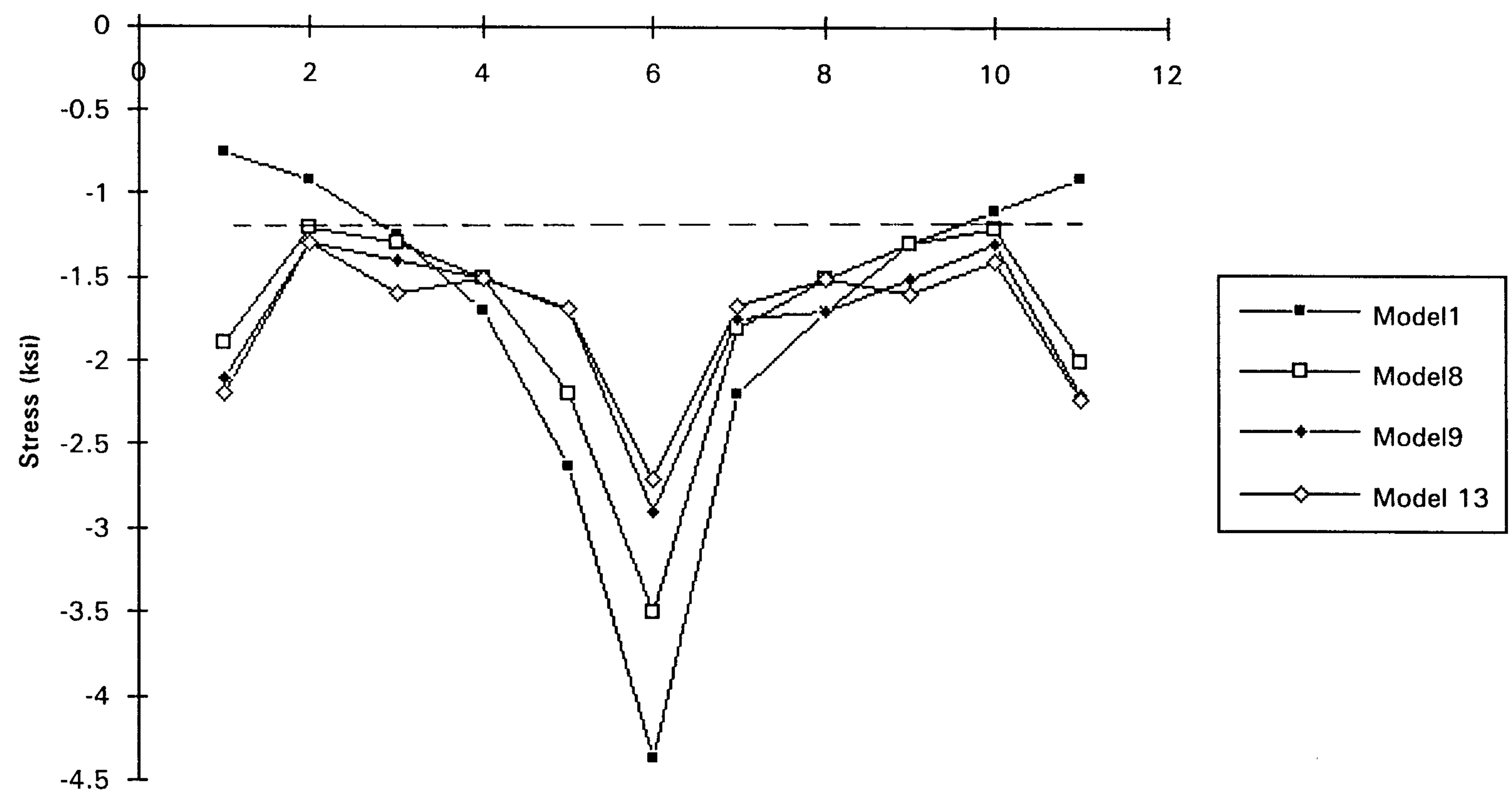


Figure 13. Stiffener Models: Top Flange Stress Results at Groove Weld

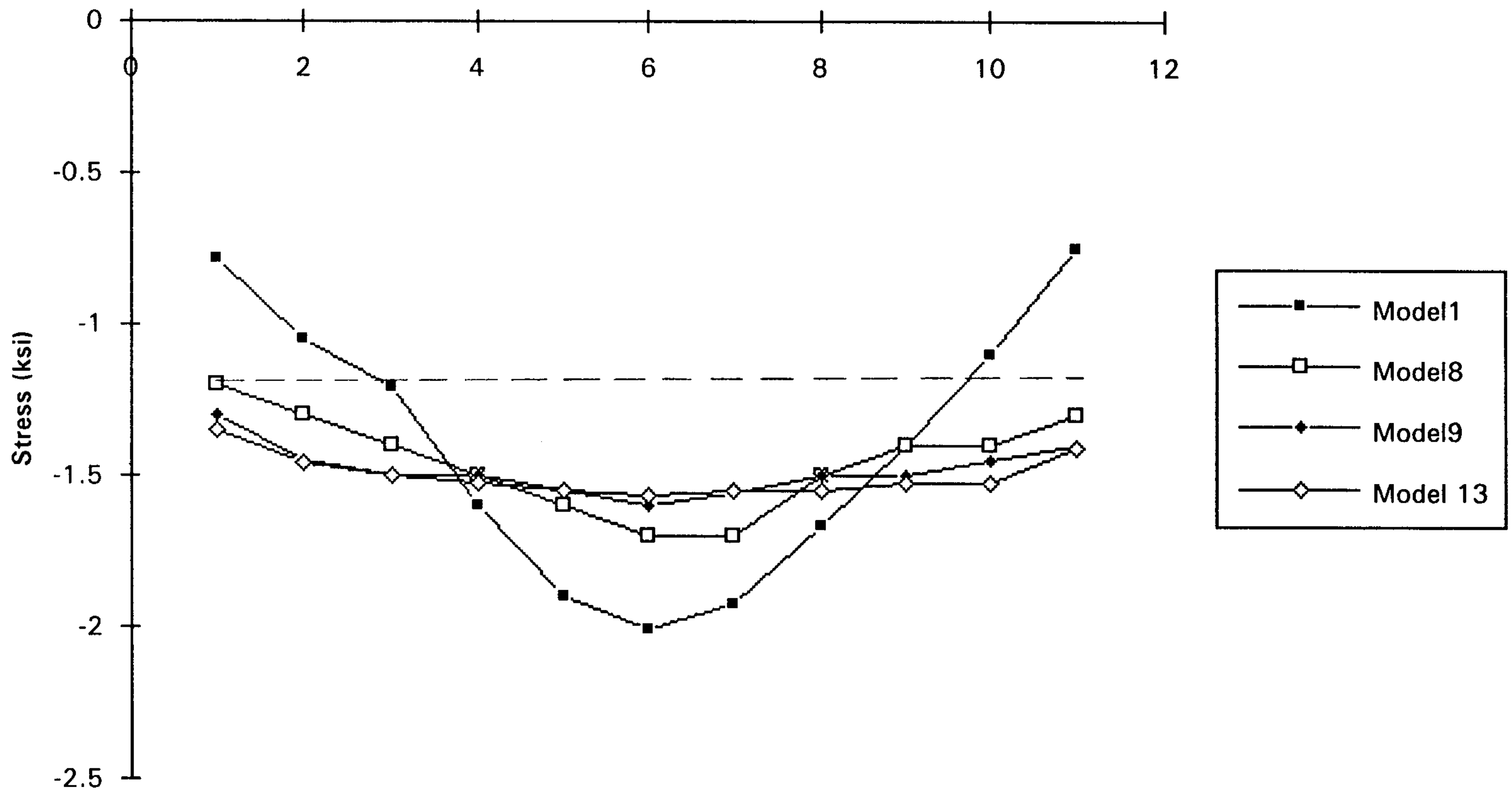


Figure 14. Stiffener Models: Top Flange Stress Results at 1.5" from Groove Weld

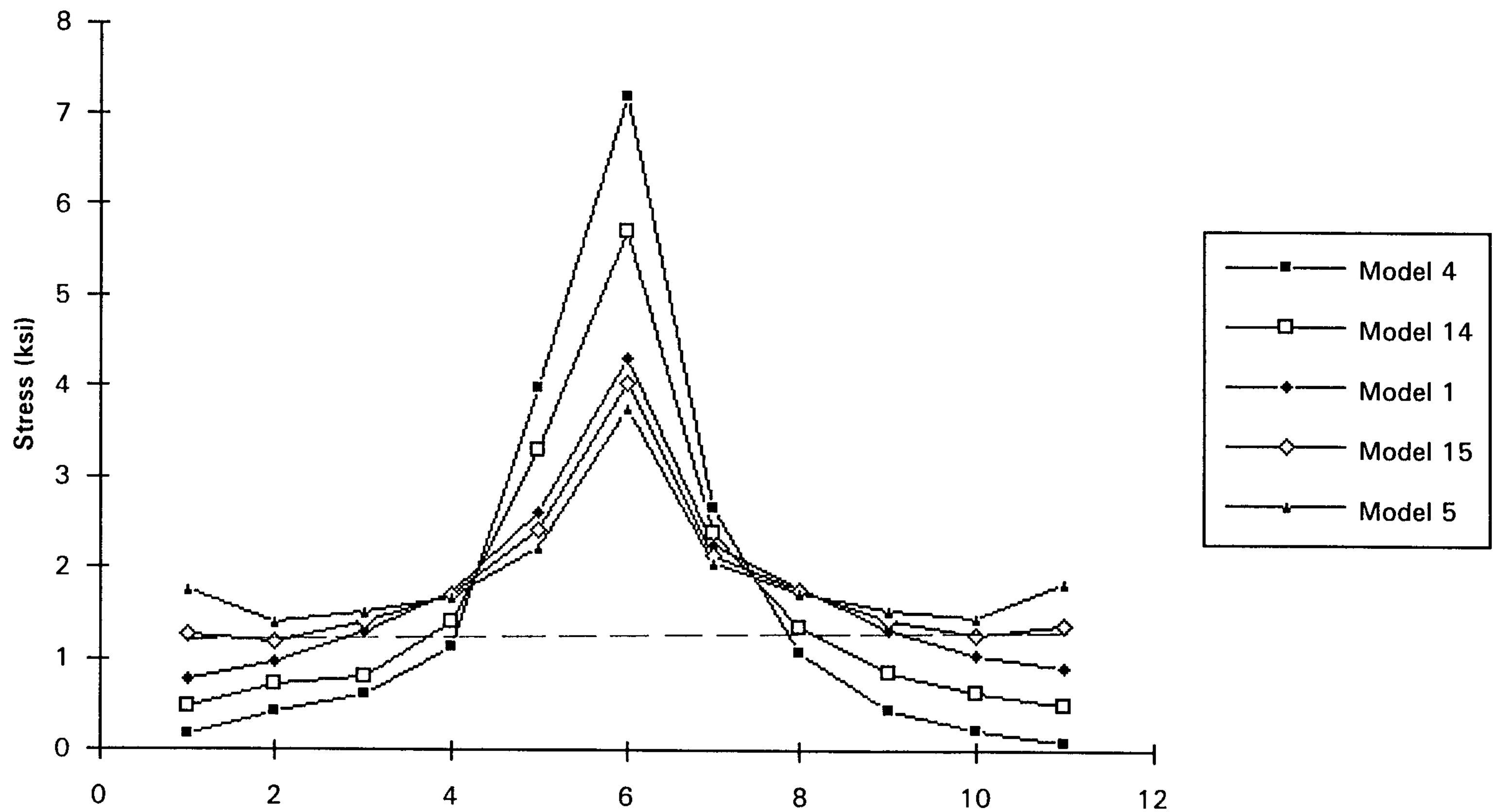


Figure 15. Column Flange Models: Bottom Flange Stress Results at Groove Weld

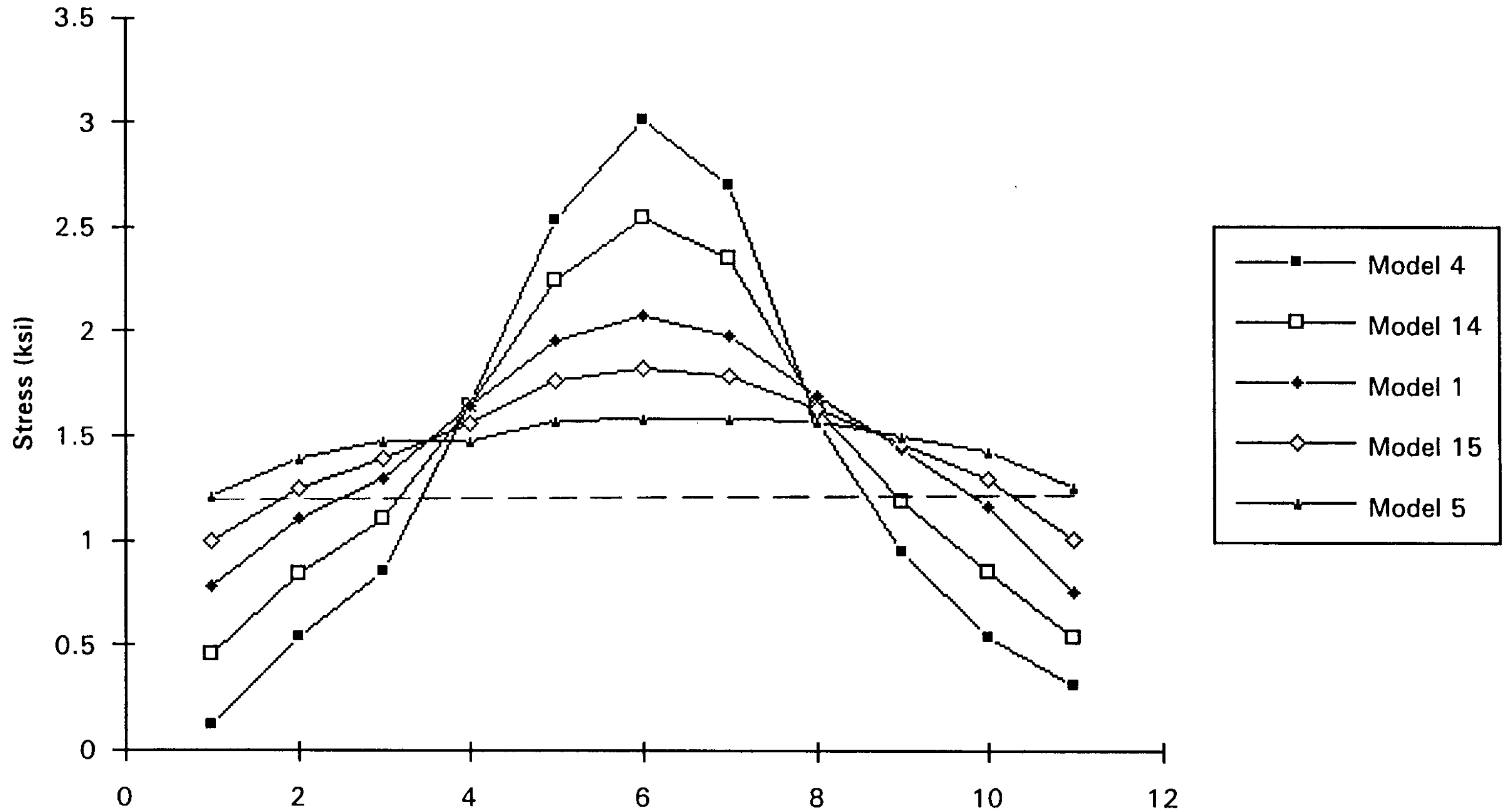


Figure 16. Column Flange Models: Bottom Flange Stress Results at 1.5" from Groove Weld

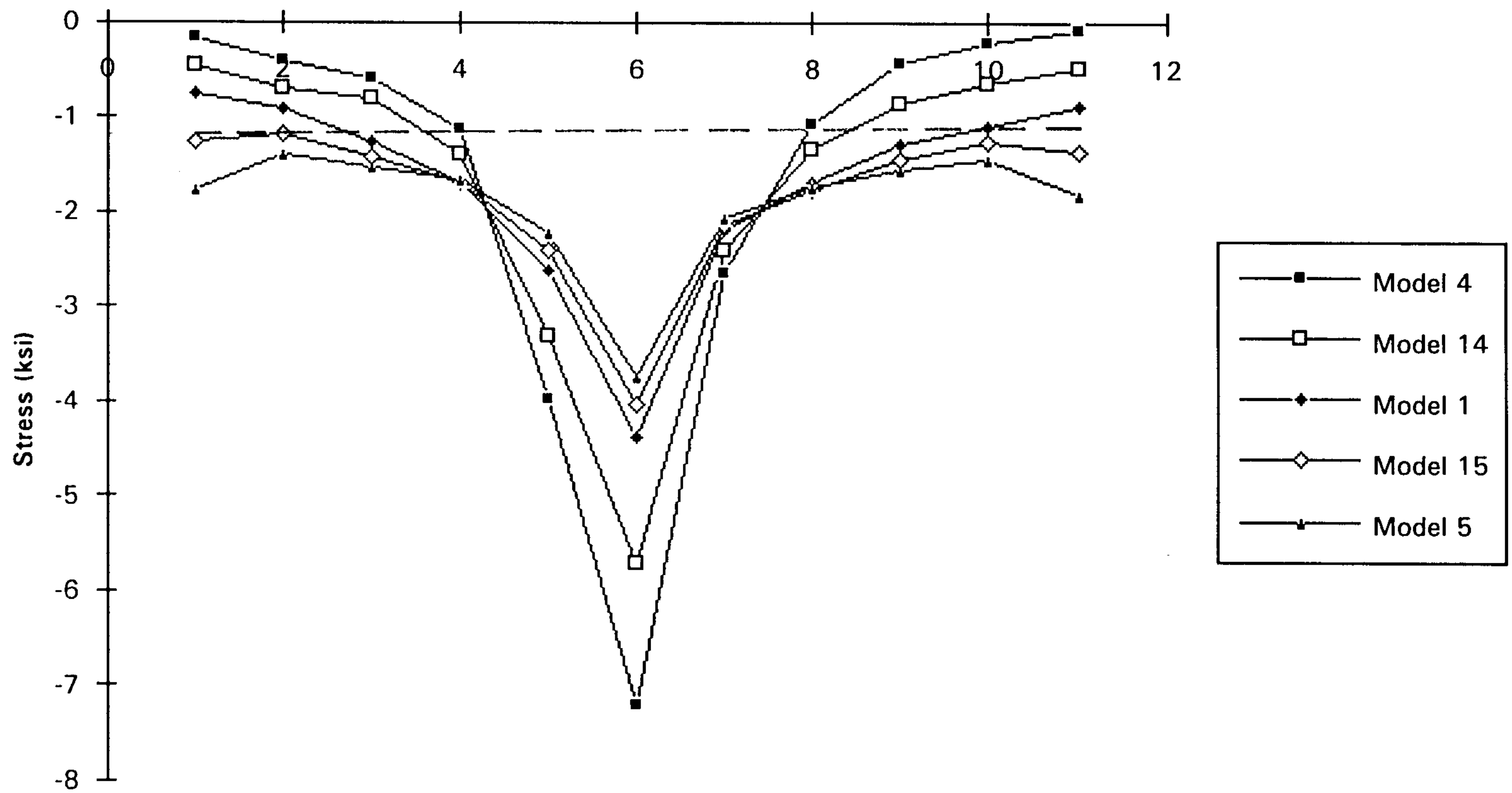


Figure 17. Column Flange Models: Top Flange Stress Results at Groove Weld

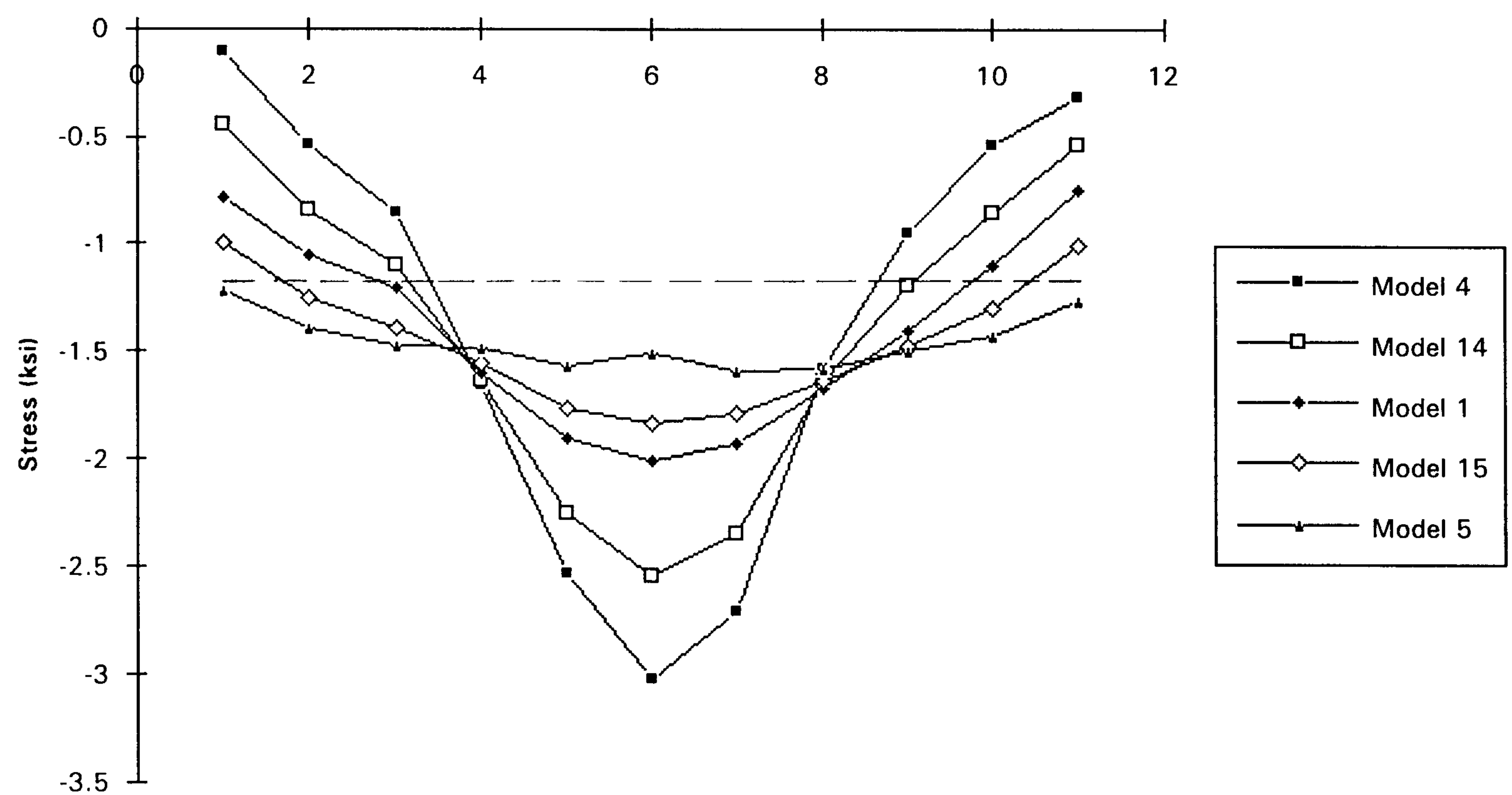


Figure 18. Column Flange Models: Top Flange Stress Results at 1.5" from Groove Weld

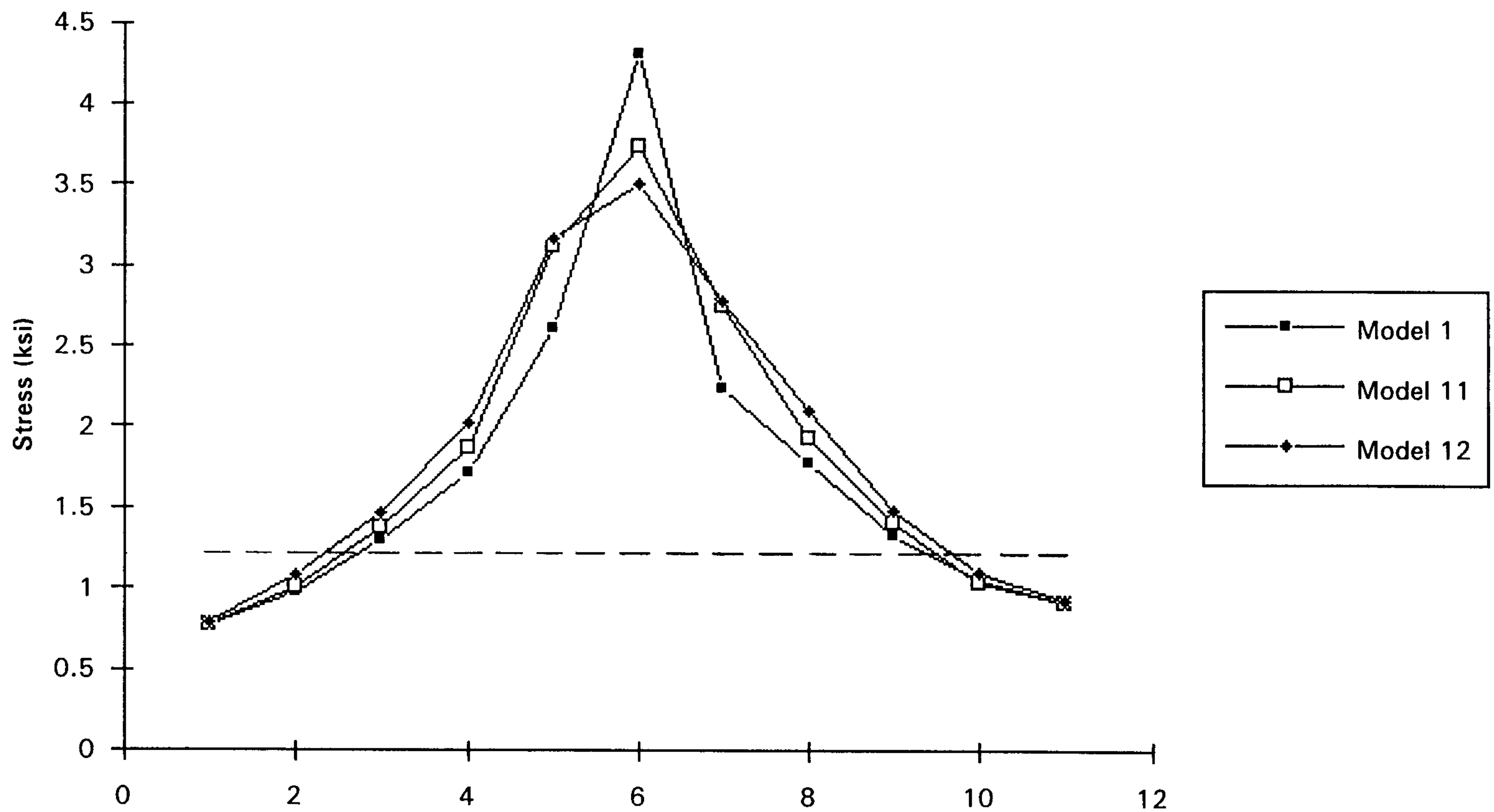


Figure 19. Beam Cope Models: Bottom Flange Stress Results at Groove Weld

Chapter 5: Summary and Conclusions

Fifteen SAP90 computer models were developed and tested in this study to facilitate investigation into the effect of various joint parameters on the stress distribution of a welded flange-bolted web connection. Parameters varied within these models were the column stiffener thickness, column flange thickness, column web thickness, and beam cope size.

Column stiffener thickness was found to have a significant effect on the stress concentration at the interface of the beam and column flanges. In models with stiffener thickness equal to that of the beam flange, stresses were found to be 33% less than the original model where no stiffeners were used.

Column flange thickness was found to have the most pronounced effect on stress concentrations. Models with flange thicknesses 25% less than the original model were found to have stress concentrations 19% higher than the original model. However, a flange thickness 50% greater than the original resulted in a stress drop of only 4%.

Column web thickness was found to have very little effect on the stress concentration. One should remember that the stresses under investigation were stresses normal to the plane of the column flange. It has been shown that web thickness has a significant effect on the shear stresses found in the panel zone [1].

Size and geometry of the beam cope was found to have a relatively small impact on the stress concentrations. However, it has been shown that variability in the preparation and execution of the full penetration groove welds, which includes preparation of the beam copes, has a significant impact on overall joint performance [10]. Furthermore, welder experience and proficiency as well as construction site conditions and practices are highly variable and their impact is not fully realized nor can it readily be accounted for by structural engineers.

Symmetry of top and bottom flange results verified in yet another way, the accuracy and appropriateness of the model but it was not possible to correctly account for the restraint provided by the concrete floor slab. For this reason as well as the previous discussion of groove weld variability, experimental investigation is necessary to further illustrate these flange stress concentrations.

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Appendix A: SAP90 Input File for Model 1

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MODEL 1 - WELDED FLANGE BOLTED WEB MOMENT RESISTING CONNECTION
C This is file MODEL1 written by SAPIN on Wed Nov 16 09:52:43 1994
C Units are KIP INCHES
SYSTEM
 R=0 L=1 C=0 V=0 T=0.0001 P=0 W=0 Z=0
GRID
 XN=1 YN=2 ZN=2
 12.16
 0 46.13
 0 0.4535
JOINTS
 7    X=0 Y=0 Z=-6.2
 85   X=0 Y=46.13 Z=6.2
 91   X=0 Y=46.13 Z=-6.2
 1    X=0 Y=0 Z=6.2 Q=1,7,85,91,1,7
 110  X=12.16 Y=0 Z=-6.2
 320  X=12.16 Y=46.13 Z=6.2
 338  X=12.16 Y=46.13 Z=-6.2
 92   X=12.16 Y=0 Z=6.2 Q=92,110,320,338,1,19
 342  X=9.728 Y=0 Z=0
 387  X=2.432 Y=46.13 Z=0
 390  X=9.728 Y=46.13 Z=0
 339  X=2.432 Y=0 Z=0 Q=339,342,387,390,1,4
 401  X=12.66 Y=11.533 Z=-3.503
 490  X=17.16 Y=11.533 Z=3.503
 500  X=17.16 Y=11.533 Z=-3.503
 391  X=12.66 Y=11.533 Z=3.503 Q=391,401,490,500,1,11
 511  X=12.66 Y=34.598 Z=-3.503
 600  X=17.16 Y=34.598 Z=3.503
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 501  X=12.66 Y=34.598 Z=3.503 Q=501,511,600,610,1,11
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1119  JQ=380,1103,384,1104 ETYPE=0 M=1 TZ=0 TH=0.79,0.79 LP=0 G=1,1
1120  JQ=384,1104,388,1105 ETYPE=0 M=1 TZ=0 TH=0.79,0.79 LP=0 G=1,1
1121  JQ=1093,341,1094,345 ETYPE=0 M=1 TZ=0 TH=0.79,0.79 LP=0 G=1,1
1122  JQ=1094,345,1095,349 ETYPE=0 M=1 TZ=0 TH=0.79,0.79 LP=0 G=1,1
1123  JQ=1095,349,1096,353 ETYPE=0 M=1 TZ=0 TH=0.79,0.79 LP=0 G=1,1
1124  JQ=1096,353,1097,357 ETYPE=0 M=1 TZ=0 TH=0.79,0.79 LP=0 G=1,1
1125  JQ=1097,357,1098,361 ETYPE=0 M=1 TZ=0 TH=0.79,0.79 LP=0 G=1,1
1126  JQ=1098,361,1099,365 ETYPE=0 M=1 TZ=0 TH=0.79,0.79 LP=0 G=1,1
1127  JQ=1099,365,1100,369 ETYPE=0 M=1 TZ=0 TH=0.79,0.79 LP=0 G=1,1
1128  JQ=1100,369,1101,373 ETYPE=0 M=1 TZ=0 TH=0.79,0.79 LP=0 G=1,1
1129  JQ=1101,373,1102,377 ETYPE=0 M=1 TZ=0 TH=0.79,0.79 LP=0 G=1,1
1130  JQ=1102,377,1103,381 ETYPE=0 M=1 TZ=0 TH=0.79,0.79 LP=0 G=1,1
1131  JQ=1103,381,1104,385 ETYPE=0 M=1 TZ=0 TH=0.79,0.79 LP=0 G=1,1
1132  JQ=1104,385,1105,389 ETYPE=0 M=1 TZ=0 TH=0.79,0.79 LP=0 G=1,1

RESTRAINTS
1099 1099 1   R=1,0,1,0,0,0
910 910 1   R=0,1,1,0,0,0
1093 1093 1   R=0,1,1,0,0,0
1105 1105 1   R=0,0,1,0,0,0
765 875 110  R=0,0,1,0,0,0

CONSTRAINTS
1056 1056 1   C=616,616,616,0,0,0 I=0,0,0,0,0,0
1058 1058 1   C=965,965,965,0,0,0 I=0,0,0,0,0,0
1060 1060 1   C=983,983,983,0,0,0 I=0,0,0,0,0,0
1062 1062 1   C=992,992,992,0,0,0 I=0,0,0,0,0,0
1064 1064 1   C=646,646,646,0,0,0 I=0,0,0,0,0,0
1066 1066 1   C=1019,1019,1019,0,0,0 I=0,0,0,0,0,0

LOADS
1  7  6   L=1 F=0,0.85294,0,0,0,0
2  3  1   L=1 F=0,1.7069,0,0,0,0
5  6  1   L=1 F=0,1.7069,0,0,0,0
4  4  1   L=1 F=-0.097564,2.2031,0,0,0,0
92  110  18  L=1 F=0,0.089355,0,0,0,0
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93   100  1    L=1 F=0,0.17871,0,0,0,0
102  109  1    L=1 F=0,0.17871,0,0,0,0
101  101  1    L=1 F=-0.097564,0.08789,0,0,0,0
339  339  1    L=1 F=-0.19513,1.0101,0,0,0,0
340  340  1    L=1 F=-0.14635,0.68371,0,0,0,0
341  341  1    L=1 F=-0.14635,0.56629,0,0,0,0
342  342  1    L=1 F=-0.19513,0.65788,0,0,0,0
1093 1093 1    L=1 F=-0.097564,0,0,0,0,0
85   91   6    L=1 F=0,-0.28806,0,0,0,0
86   87   1    L=1 F=0,-0.57712,0,0,0,0
89   90   1    L=1 F=0,-0.57712,0,0,0,0
88   88   1    L=1 F=0.097564,-0.4934,0,0,0,0
320  338  18   L=1 F=0,-0.29164,0,0,0,0
321  328  1    L=1 F=0,-0.58329,0,0,0,0
330  337  1    L=1 F=0,-0.58329,0,0,0,0
329  329  1    L=1 F=0.097564,-1.0866,0,0,0,0
387  387  1    L=1 F=0.19513,-0.68888,0,0,0,0
388  388  1    L=1 F=0.14635,-0.58929,0,0,0,0
389  389  1    L=1 F=0.14635,-0.70671,0,0,0,0
390  390  1    L=1 F=0.19513,-1.0411,0,0,0,0
1105 1105 1    L=1 F=0.097564,-0.432,0,0,0,0
760  770  10   L=1 F=0.17121,0,0,0,0,0
761  764  1    L=1 F=0.34241,0,0,0,0,0
766  769  1    L=1 F=0.34241,0,0,0,0,0
765  765  1    L=1 F=1.3186,0.052741,0,0,0,0
870  880  10   L=1 F=-0.17121,0,0,0,0,0
871  874  1    L=1 F=-0.34241,0,0,0,0,0
876  879  1    L=1 F=-0.34241,0,0,0,0,0
875  875  1    L=1 F=-1.3186,0.052741,0,0,0,0
890  890  1    L=1 F=0.65076,0.10548,0,0,0,0
900  900  1    L=1 F=0.32538,0.10548,0,0,0,0
920  920  1    L=1 F=-0.32538,0.10548,0,0,0,0
930  930  1    L=1 F=-0.65076,0.10548,0,0,0,0
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